Bridge Loads

1.0.1 General

Loads are fundamental to bridge design, having evolved with experience and study over many years. They have been codified in the United States since the mid-1920s in the Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO). In Caltrans, these design specifications are contained in the Bridge Design Specifications (BDS). BDS is indexed to correspond with AASHTO, Division 1, Design. It includes some non-AASHTO material selected in anticipation of future AASHTO adoption or because of its local importance.

This section, Bridge Loads, deals with general aspects of the loads specified in the Thirteenth Edition of BDS, Section 3, Loads. The material covers load definitions; requirements and practices regarding distribution of load effects; adjustment of raw loads by specified load factors; and combination of loads with one another into specified groups acting together. In addition, it is prefaced by a brief introduction to Load Factor Design. This section may be considered a commentary on BDS Section 3.

The specific uses of loads in the context of design processes are illustrated in the text sections dealing with structure elements and construction materials.

1.0.2 Design Methods

AASHTO provides specifications suitable for two distinct design methods, Load Factor Design (LFD) and Service Load Design (SLD). Caltrans' policy is to use LFD to the greatest extent possible. Therefore it receives the bulk of attention in BDS and in this text.

SLD, until recent years, was the primary design method upon which AASHTO was based. It has been known historically also as working stress design or elastic design. Its main objective is to equate load effects with allowable stress, a specified fraction of the yield strength of steel or ultimate strength of concrete. Its factor of safety against failure is implicit in that fraction. It draws on elastic theory for its fundamental concepts. SLD does not consider structural performance beyond the elastic range.

LFD implementation in Caltrans has been in progress since 1974. There are still a few lingering exceptions to its use, but it is now firmly established as the basic Caltrans design method.

The main objective in LFD is to equate ultimate load-carrying capacity with applied loads, after both have been modified by safety factors. Nominal or theoretical ultimate capacity, with stresses at the verge of failure (yield point of steel), is reduced by a materials confidence factor. Applied loads are adjusted by multipliers of both the individual loads and the combinations of loads acting together.

The net effect is calculated to maintain stresses usually within the elastic range. LFD follows ultimate strength theory as well as elastic theory for its fundamentals.

LFD was adopted by Caltrans as much for its consequences after construction as for its refinements in logic and precision during design. Because its safety factors relate directly to loads as well as materials, rather than just to materials as in SLD, it is possible to design bridges with consistently uniform usable live loads allowed on the highway system by special permit. In LFD, usable live load is clearly and directly represented by design live load.

In SLD, design capacity is based on computed loads with a prescribed safety factor which is the same for dead and live loads. Capacity for permit live loads is based on higher stresses (lower safety factor) than those used for design.

This shift in stress levels provides usable live load capacity from two sources; from capacity originally provided for live load, increased now by higher allowable stresses; and from a similar increase in dead load capacity, which is not needed to support added dead load. The ratio of dead to live load in a structure varies markedly from one structure type to another. It is relatively high for concrete; low for steel structures.

The use of excess dead load capacity as a source of live load capacity in bridges designed by SLD has resulted in a disorderly variation of permit load capacity from bridge to bridge along stretches of highway with mixed structure types. The bridge with the least permit capacity controls, thus preventing the use of available additional capacity in the rest of the group. The use of LFD avoids this problem.

1.1 Load Definitions

1.2 Dead Loads

Dead loads consist of the weight of permanent portions of the structure, including the effects of anticipated future additions.

Designs must provide for an additional 35 pounds per square foot dead load for future deck overlay. Long ramp connectors and special major structures in regions of mild climate are exempt from this requirement.

The effects of future utilities and planned future widenings need special attention to assure they are accommodated in the design.

1.3 Earth Pressure On Culverts

Research has indicated that the earth weight to be used in design of culverts shall be as modified in BDS 6.2 in order to provide sufficient strength.

More information and references are contained in BDP Section 6, STRUCTURES UNDER ROADWAY EMBANKMENTS; BDS Section 6, CULVERTS.

1.4.1 Live Loads

Bridges on the State Highway System are subjected to a variety of live loadings, including vehicular, equestrian, pedestrian and others.

This discussion is limited to highway vehicle loads, which are divided into three load systems: H loads, Alternative loads and P loads. These loads are shown in Appendixes A.1 and A.2 and the effects of moment and shear are compared in Appendix A.3.

The example problems that follow in Appendixes A.4 and A.5 demonstrate the application of H loads to a continuous bridge superstructure in order to obtain controlling conditions for design. It is assumed that the reader has knowledge of structural mechanics which will enable him to make the necessary computations. Solution by computer greatly expedites the work. The Office of Structure Design makes extensive use of the computer program, Bridge Design System, because it is tailor-made for the purpose.

1.4.2 H Loads

The H and HS trucks are live loads used in bridge design to ensure a minimum load carrying capacity. These loads represent a vast number of actual truck types and loadings to which the bridge might be subjected under actual traffic conditions. They are the original AASHTO design live load system, dating from the 1920s. They have been revised and expanded periodically since then, but still retain their original character.

The lane load is a simplified loading which approximates a 20-ton truck preceded and followed by 15-ton trucks.

For simple spans, one truck is the governing H load for moment in spans less that 145 feet, and the lane load governs for longer spans.

In continuous spans, the lane loading governs the maximum negative moment, except for spans less than about 45 feet in length where the HS truck loading with its 32 kip axles, variably spaced from 14 feet to 30 feet, may govern. The exact point of change of controlling load depends on ratios of adjacent span lengths. The positive moment of continuous spans is usually controlled by the lane loading for spans of more than about 110 feet.

1.4.3 Alternative Loads

The basic alternative loading consists of two axles spaced four feet apart with each axle weighing 24 kips. This load produces slightly greater live load moments than H loads in spans under 40 feet.

The Alternative Load originated as a Federal Highway Administration requirement for bridges on the Interstate Highway System in 1956. It provides capacity for certain heavy military vehicles. For convenience and uniformity, it is applicable to the design of all bridges in the State Highway System.

In addition, a single 32 kip axle with the weight equally divided between two wheels centered six feet apart is used in design of transversely reinforced bridge decks.

For discussion purposes, alternative loads are frequently combined with H loads and referred to jointly as H loads, in contrast to P loads.

1.4.4 P Loads

P loads are special design vehicles in addition to the H loads and Alternative loads specified by AASHTO. The P loads were developed in California to ensure sufficient live load capacity to carry extralegal live loads allowed by permit.

Permit design loads (P loads) consist of a family of idealized vehicles (see Appendix A.2) used by Office of Structure Maintenance and Investigations in rating bridge capacities. The steering axle and any number from two to six pairs of tandems (assumed as single concentrated loads) constitute a valid design vehicle. The combination that produces the maximum effect is used.

These loads were adopted for design in Caltrans because without them the AASHTO provisions for LFD would, in many cases, result in structures incapable of carrying permit loads in actual use or anticipated on California highways.

1.4.5 Highway Vehicle Traffic Lanes

The basic highway vehicle load width is 10 feet, which applies to all design trucks, lane loads and axles.

Virtually all design lanes are 12 feet wide.

The 10-foot wide loads are allowed to move within the 12-foot wide lanes which, in turn, may move between the curbs. The number of loads, their positions within the lanes, and the location of the lanes themselves are as required to produce maximum effects in the member under consideration.When applying trucks to determine maximum effects in a member, only one truck per lane is utilized.

Fractional parts of lanes are not allowed for bent caps and substructure and members.

Live load reduction factors are applicable to substructure members and some superstructure members. These factors represent the probability that several lanes of full design load will not occur simultaneously on the bridge.

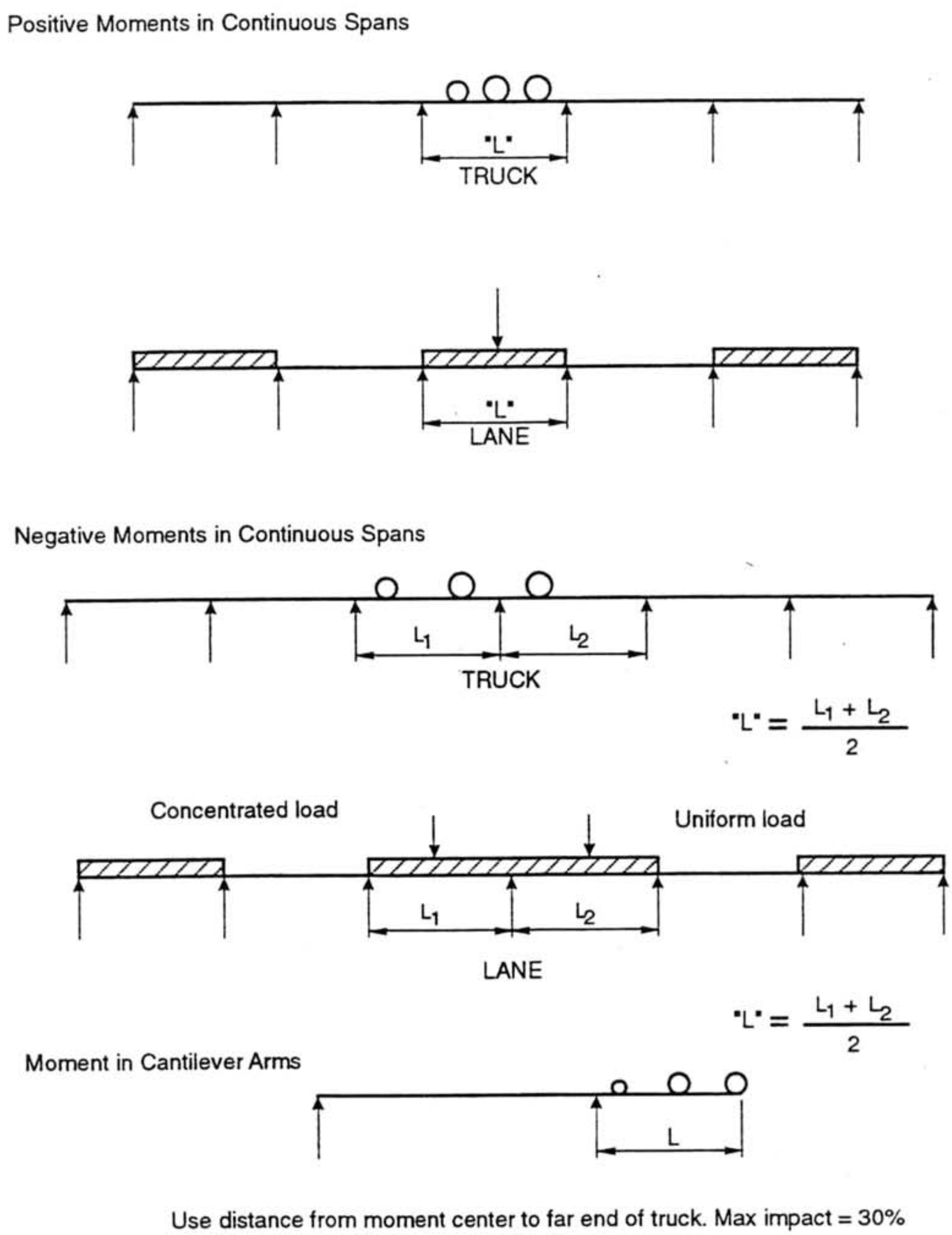
1.4.6 Highway Vehicle Load Application

The basic live load design objective is to satisfy both H load and P load requirements. Structural components are proportioned for these loads at either the factored level or service level of magnitude as specified for the structural material or system under consideration.

Correction for sidesway is not normally made for live load because the duration of the loading is not long enough for sidesway to occur.

1.5 *Impact*

Impact is added to live loads for most structural members which are above ground to account for the dynamic effect of these loads. However, impact is not added to loads on timber members. Following are some illustrations of the loaded length, "L" for use in the impact formula for highway vehicles.



Impact Examples
Figure 1

1.6 Longitudinal Force

Provision is made for the longitudinal traction and braking effects of vehicular traffic headed in the same direction in Load Combination Groups III and VI only (BDS 3.22). The longitudinal force of P loads is not considered.

The longitudinal force, when combined with the other forces, may affect the design of bents. Occasionally, in rigid frame structures where the bents are very stiff, longitudinal force, when added to other forces, may affect superstructure design.

The application of the force six feet above the roadway does not change girder moments much. We are more concerned with longitudinal force as a shear on the column tops.

The specifications describe friction effects due to various types of expansion bearings. The friction forces are transmitted to the substructure as reactions from horizontal design forces on the superstructure. However, friction is not an independent primary force that requires consideration for group action in BDS 3.22.

1.7.1 Wind Load On Structures

Wind loads are applied to all structures and highway vehicle live loads except P loads.

The basic wind loads result from a high wind of 100 mph and a moderate wind of 30 mph. In general, the high wind is assumed to act on the structure when live load is *not* present. Moderate wind acts on the structure when live load *is* present, for some load combinations.

The basic high wind of 100 mph produces 75 psf on arches and trusses, 50 psf on girders and beams, and 40 psf on substructures. When Load Combination Groups III and VI are considered, a moderate wind of 30% of the high wind pressure is used.

This force is applied in a variety of ways depending on whether one is designing superstructure or substructure, and whether the structure is usual or unusual.

Horizontal wind loads on the superstructure are always based on the area seen in elevation view. They act both longitudinally and transversely. Loads on the substructure can be applied to elevation or transverse views, or skew angles in between.

When calculating the forces tending to overturn a structure, the upward high wind pressure of 20 psf (based on plan view area) is used for Groups II, V and IX, while the moderate wind pressure of 6 psf is used for Groups III and VI.

The specifications provide for the use of judgement concerning wind velocities to be used in structure design. Permanent terrain features or precise data from local weather service records may indicate that the basic 100 mph design wind should be modified. If such is the case, the specified wind pressure is changed in the ratio of the square of the design wind velocity to the square of 100. Whenever this is done, the revised design wind must be stated in the General Notes of the bridge plans.

For high structures the wind effects on the bents and footings need to be thoroughly investigated, both laterally and longitudinally.

The limiting height of column where wind may control varies with the span length, physical makeup of the structure, and the magnitude of other lateral loads such as that due to earthquakes.

When applying lateral loads in continuous structures, consideration should be given to the rigidity of the deck and its ability to transfer wind loads to abutments which might be considerably stiffer than the bents. In these cases, the abutments must be designed to support these lateral loads.

1.7.2 Wind On Live Loads

In addition to moderate wind pressure on the bridge structure when live load is present, a moderate wind force is exerted on the live load itself. This force is expressed as a line load acting both transversely and longitudinally 6 feet above the roadway surface. This offset location is not important except perhaps for the design of high piers.

1.8 Thermal Forces, Shrinkage and Prestressing

Structural members are investigated to satisfy the Design Range of temperatures given in BDS 3.16 or the Bridge Preliminary Report. The design range provides for movement corresponding to about one-half or less of the full air temperature range. Forces due to temperature movement can become large on short stiff multicolumn bents but are usually reduced by distribution through the bent frame.

Expansion joint movement ratings are calculated to provide for the full air temperature range with allowance for creep and shrinkage. Special instructions are included in Memo to Designers 7-10.

Shrinkage is the volume decrease which occurs when fresh concrete hardens and for a period of time thereafter. It is important in arches, where rib shrinkage produces rib and column moments, and prestressed girders, where shrinkage produces loss of stressing force.

Provision for influence of movement and bending effects caused by prestressing is described in Section 3 of this manual, PRESTRESSED CONCRETE. Hinge location and substructure design are sometimes determined by prestressing effects.

1.9 Uplift Forces

Certain combinations of loading tend to lift the bridge superstructure from the substructure. Elements of the bridge must be tied together to resist these uplift forces. This can be accomplished by either providing tension ties or by providing sufficient mass in the superstructure to resist the uplift force. Uplift can become important with unusual span configurations. For instance, a very short end span adjacent to a long span will tend to lift at the abutment.

In LFD we must provide a resisting force sufficient to balance uplift caused by any load combination in BDS 3.17. For service load checks, calculated uplift force is amplified by factors to ensure safety.

1.10 Forces of Stream Current, Floating Ice and Drift

Columns and piers in streams are designed to resist the forces of water, ice and drift. The Bridge Preliminary Report will describe requirements for these items, when necessary. Piers should be located and skewed to afford minimum restriction to the waterway as recommended in BDS Section 7, SUBSTRUCTURES.

Box girders or slabs are recommended superstructure types where less than 6 feet of clearance is provided over a stream carrying drift.

1.11 Buoyancy

Whenever a portion of a structure will be submerged, the effects of buoyancy should be considered in the design. In small structures, its effects are unimportant and no economical advantage can be realized in the footing design. In large structures, however, its effects should be taken into account in the design of footings, piles and piers.

1.12 Earth Pressure on Abutments and Retaining Walls

Abutments and retaining walls should be designed so that any hydrostatic pressure is minimized by providing adequate drainage for the backfill. References at the end of this section and BDP Section 5, SUBSTRUCTURES AND RETAINING WALLS, include more information on the application of soil mechanics to abutment and retaining wall design.

Symbols used in BDS 3.20 are:
K_a = active earth pressure coefficient
w = unit weight of soil (pounds per cubic foot)
h = height (feet)
S = live load surcharge height (feet)

For level backfill, the minimum active earth pressure is usually taken as an equivalent fluid pressure of 36 pounds per square foot per foot of height for abutments and retaining walls. This is based on an earth pressure coefficient (K_a) of .30 and a unit weight (w) of compacted earth of 120 pounds per cubic foot. This is used in design of following elements:

(1) Toe pressure or toe piles in retaining walls and abutments.
(2) Bending and shear in retaining walls and abutments.
(3) Sliding of spread footings or lateral loads in piles.

For the design of rear piles in retaining walls or abutments, checks should be made using an equivalent fluid pressure of 27 pounds per square foot per foot of height. This corresponds to a K_a of .225.

A trapezoidal pressure distribution is used where the top of wall is restrained. This provides a more realistic solution than the triangular pressure distribution which applies to typical retaining walls without restraint.

1.13 Seismic Force

Earthquakes and the response of structures to earthquakes, are dynamic events—events that go into many cycles of shaking. An earthquake of magnitude 8+, such as that which occurred in San Francisco in 1906 and in Alaska in 1964, may have strong motions lasting for as long as 40 to 60 seconds. The San Fernando earthquake of magnitude 6.6 had about 12 seconds of strong motion.

During this period of strong motion, the structure passes through many cycles of deflection in response to the motions applied at the base of the structure. The strains resulting from these deflections are the cause of the structural damage.

Structures subjected to earthquake forces shall be designed to survive the strains resulting from the earthquake motion. Factors that are considered when designing to resist earthquake motions are:

(1) The proximity of the site to known active faults.
(2) The seismic response of the soil at the site.
(3) The dynamic response characteristics of the total structure.

The foundation report prepared by the Caltrans Division of New Technology Materials and Research, Office of Engineering Geology, contains the seismic information necessary for design.

Three methods of analysis are available to design structures to resist earthquake motions. They are the Equivalent Static Force Method; Response Spectrum Modal Analysis; and Time History Method.

Equivalent Static Force Method

Column and member forces may be calculated using the equivalent static force method of analysis. It was developed as a simple way to design for the strains associated with earthquake motions and is suitable for hand calculations. This approach is effective when the mode shape (deflected shape under vibration) can be approximated in each direction being analyzed and when one mode dominates in each direction.

The method assumes a predominant deflected shape and location of maximum displacement when vibrated in the direction under consideration.

Curves are used which consider this period, the depth of alluvium under the structure, and the expected maximum acceleration of bedrock based on geology of the site. A value for the seismic coefficient is determined from these curves which represent the elastic response of the bridge to the earthquake. This value is then used to determine the maximum displacement in the structure. Design forces in ndividual members are then computed for the displacement.

Structures with no more than one intermediate hinge and having the following characteristics may be analyzed using the equivalent static force method:

A. Tangent or nearly tangent alignment.
B. Total deck length to width ration less than 15.
C. Skew angles of abutments and bents less than twenty degrees.
D. Balanced spans and supporting bents or piers of approximately equal stiffness.

Response Spectrum Modal Analysis

The response spectrum technique of modal analysis should be considered for determining earthquake loads when the bridge does not fall into the categories listed above. In this case, several modes of vibration will probably be significant contributors to the overall seismic response of the structure.

This method of analysis is computer oriented. The amount of calculation necessary makes it impractical, if not impossible, to do by hand. The computer first determines all modes of vibration that a three dimensional mathematical model of the structure can have. It then applies a response spectrum loading using the same curves used for the equivalent static force method. These loadings are applied to the structure for each mode of vibration. The computer reports the deflections and forces thus induced both for each mode and for the root mean square summation of all modes.

Time History Method

Time history analyses should be utilized for unusual structures. Structures for consideration have sites adjacent to active faults, sites with unusual geologic conditions, unique features, or a fundamental period greater than 3.0 seconds. They are usually large, complex and important structures.

This computer analysis is the most complex (and expensive) of the three methods. The computer actually subjects a mathematical model of the structure to an idealized earthquake. It does this by subjecting the computer model to earthquake impulses at predetermined time intervals. These intervals are in small fractions of a second representing the ground accelerations varying with time. The forces in the various parts of the structure can either build or cancel under these impulses as time passes depending on the vibration characteristics of the structure. The computer reports these forces at their maximum and for any desired point in time. This method gives the designer the best understanding of the true dynamic characteristics of earthquake loading.

All of these methods are based on elastic theory. None of them is capable of modeling structural behavior when material strains are in the inelastic range. In a major earthquake, structural movement will almost surely be in the inelastic range. Recognizing this fact and realizing that it is impractical of design bridges to behave elastically under attack from a large earthquake, a ductility and risk factor, Z, is introduced. The seismic forces that are produced by any of the described methods of analysis are divided by this factor before design. The value of Z varies depending on an assessment of the ability of a particular bridge member to withstand strain in the inelastic range as well as the member's importance in preventing collapse. See Figure 3.21.1.2 in BDS 3.21.

1.14 Centrifugal Force

Centrifugal forces are included in all groups which contain vehicular live load, including P loads. They act 6 feet above the centerline of roadway surface.

Centrifugal forces are significant in the design of bridges having small curve radii or curved bridges with long columns.

These forces act as shears in girder end frames and as loads at tops of columns. Again, the 6-foot vertical dimension between the point of load application and the floor is seldom important.

1.15.1 Load Distribution

Load distribution is the process by which the effects (forces, moment, shear, reaction, torsion) of a load flows from the point of application to all other locations within a structure and into the foundation.

Distribution of most loads is accomplished through rational analysis by some accepted mathematical method. Statistics, moment distribution and stiffness analysis are common forms of rational analysis.

The most important exception to this approach is contained in the provisions of BDS 3.23 for vehicular live load and certain aspects of dead load distribution. These simplifying assumptions and empirical formulae are specified for convenience and uniformity. They reflect the results of research and the state of the art current at the times they were adopted by AASHTO.

1.15.2 Dead Loads

Dead load is usually distributed to supporting members by an appropriate rational analysis. However, for simplicity, BDS 3.23.2.2 specifies that the weight of curbs, sidewalks, railings and wearing surface may generally be distributed uniformly to all stringers and beams.

1.15.3 Highway Vehicles

Slabs are loaded by individual wheels. Our design specifications are based on plate theory to find the resulting maximum design effects. Standard designs are available for transverse deck slabs on girders and for longitudinally reinforced slab bridges. See *Bridge Design Details 8-30* and *Bridge Design Aids 4-10 through 4-19.*

Bridge girders, stringers, and some floor beams are loaded by lines of wheel loads that roll along the deck. A wheel line is half of a truck or lane load. The number of lanes assigned to each girder depends on the girder spacing and type of girder.

The live loads are moved longitudinally along the bridge, and as they move, they generate changing effects in the bridge members. Refer to Appendices A.4 and A.5 for an example showing the application of H loads to a 3-span continuous structure.

The maximum moments or shears resulting from these moving loads are evaluated at the various locations by the computer program, Bridge Design System.

Specifically, in superstructure design we are concerned with the maximum live load effect that any one member can experience regardless of the number of live load lanes the bridge can accommodate. This effect depends on the transverse stiffness of the superstructure—its ability to distribute loads laterally. For instance, a concrete box girder distributes live load much more completely than a steel stringer bridge.

BDS 3.23 gives the mechanics for determining distribution to girders by empirical formulae that consider structure type and girder spacing. The results are generally in fractions of wheel lines per girder.

On box girder structures, the "S-over" distribution of BDS 3.23 is applied to the entire width of bridge as a unit.On structures other than box girders the "S-over" distribution applies to single girders only, where S is the girder spacing. For exterior girders, the same distribution applies but with a factor to account for length of deck overhang.

As girder spacing increases, a point is reached where the "S-over" distribution no longer applies. Beyond this point the wheel loads are distributed to the girders assuming the deck slab acts as a simple beam between girders. Limiting girder spacing for this condition depends on type of superstructure.

Because of the two criteria for wheel load distribution, depending on whether girders are "closely spaced" or "widely spaced", P load distribution for superstructure design is divided into two procedures.

(1) For closely spaced girders, P loads only are applied. Distribution is by the "S-over" formula.

    This procedure effectively applies a major P fraction to every girder in a system. The total design load on a girder system two or more lanes wide exceeds the intended single P load and adjacent H load as required in BDS 3.11. The excess capacity provided is enough to allow bonuses that exceed the P load sometimes granted with permits.

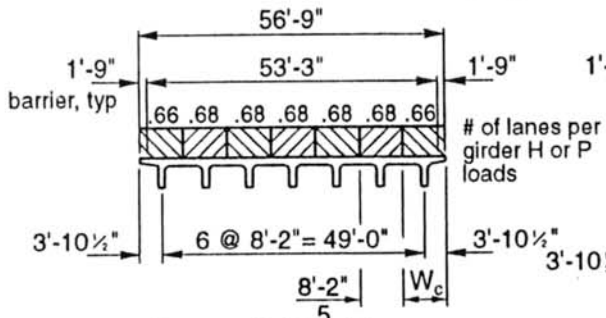
(2) For widely spaced girders, a single P load and an adjacent H load, both positioned for maximum effect are applied. Distribution is by static reactions on the girders, as specified in BDS Table 3.23.1, Footnote f.

Figure 2 shows examples of live load distribution to superstructures and substructures.

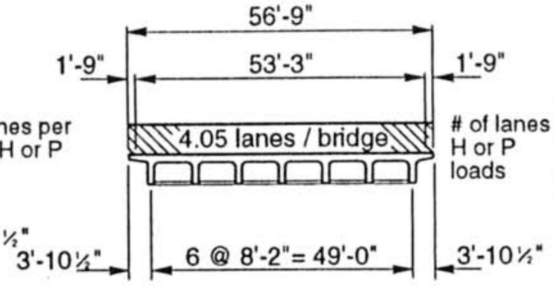
One other point to be kept in mind is the discontinuity in procedures for applying live loads to various elements of a bridge structure. The deck slab is designed according to one loading criterion, the girders by another. The live load reactions from these loads are then discarded, and we start anew as design proceeds to the substructure. Different live loads may control design at different locations. Also, the effect of live load distributes, and the effect of impact dissipates as they move down through the structure. Reductions are taken to allow for improbability that several heavy vehicles will cross a structure simultaneously. For these reasons, it is difficult to trace logically a given design live load from deck level to foundation. The total live load considered to be taken by the foundation is almost always less than that which was applied to the superstructure.

Live Load Distribution to Superstructures

Closely Spaced Girders



Concrete Tee Beam



Concrete Box Girder

Use same number of lanes for groups I_H and I_{PC} with no lane reduction factor.

Distribute by S - over formulae multiplied by ratio: $\dfrac{1 \text{ lane}}{2 \text{ wheel lines}}$

$$\frac{\text{live load lanes}}{\text{interior girder}} = \frac{S}{12}\ \frac{8.17}{12} = 0.68$$

$$\frac{\text{live load lanes}}{\text{exterior girder}} = \frac{S}{12}\left[\frac{W_C}{S}\right] = \frac{8.17}{12}\left[\frac{\frac{8.17}{2}+3.87}{8.17}\right] = 0.66 < 0.68$$

use 0.68

$$\frac{\text{total live load lanes}}{\text{bridge}} = 2(0.68)+5(0.68) = 4.76\ *$$

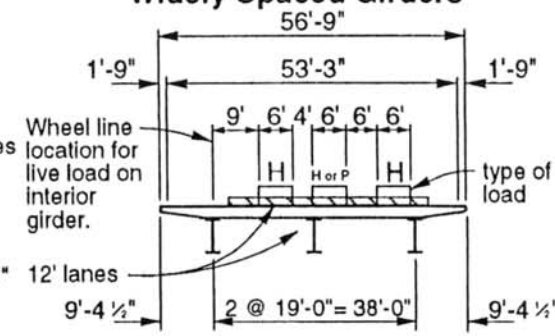
$$\frac{\text{live load lanes}}{\text{bridge}} = \frac{\text{over-all deck width}}{14}$$

$$= \frac{56.75}{14}$$

$$= 4.05$$

$$\frac{\text{live load lanes}}{\text{girder}} = \frac{4.05}{7} = 0.58\ *$$

Widely Spaced Girders



Concrete Deck on Steel Stringers

No fractional lanes allowed in determination of simple beam deck reactions.

Group I_H use max. number of lanes with one lane reduction factors. BDS 3.12

$$\frac{\text{live loads lanes}}{\text{interior girder}} = 1.52H \quad \text{(3 lanes of H loads with lane reduction factor)}$$

$$\frac{\text{live load lanes}}{\text{exterior girder}} = 1.64H \quad \text{(2 lanes of H loads)}$$

$$\frac{\text{total live load lanes}}{\text{bridge}} = 1.52H + 2(1.64)H = 4.81H\ *$$

Group I_{PW}, use 2 lanes or 1 lane

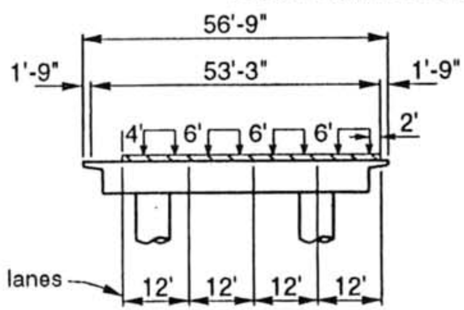
$$\frac{\text{live loads lanes}}{\text{interior girder}} = .63H + .84P \quad \text{(1 lane H load, 1 lane P load)}$$

$$\frac{\text{live load lanes}}{\text{exterior girder}} = .51H + 1.14P \quad \text{(1 lane H load, 1 lane P load)}$$

$$\frac{\text{total live load lanes}}{\text{bridge}} = 1.65H + 3.12P\ *$$

General Notes:
- Groups I_H, I_{PC} & I_{PW} are evaluated separately, not simultaneously.
- All loadings shown are un-factored.
- *Denotes values given for comparison purposes, these girders are ordinarily designed separately.

Live Load Distribution to Substructure



Example of Live Load Location

Bent caps are considered part of substructure. Use Group I_H or I_{PW}

Number of lanes and positions across roadway shall be as required to produce maximum effect.

No fractional lanes allowed.

Lane reduction factors shall apply.

For Group I_{PW}, only one P load, or one P load and one H load may be applied.

Figure 2

1.16.1 Load Factors

An essential feature of LFD, as stated in 1.0.2, Design Methods, requires raw design loads or related internal moments and forces to be modified by specified load factors (γ, gamma and β, beta), and computed material strengths to be reduced by specified reduction factor (ϕ, phi).

These are safety factors which ensure certain margins for variation. The three different kinds of factors are each set up for a distinct purpose, each independent of the other two. In this way, any one of them may be refined in the future without disturbing the other two.

1.16.2 γ (Gamma) Factor

The γ (gamma) factor is the most basic of the three. It varies in magnitude from one load combination to another, but it always applies to all the loads in a combination. Its main effect is stress control that says we do not want to use more than about 0.8 of the ultimate capacity. Its most common magnitude, 1.3. lets us use 77%. Earthquake loads are not factored above 1.0 because we recognize that stresses in the plastic range are allowed, as long as collapse does not occur.

An example may be given to justify the use of gamma of 1.3 for dead load.* Assuming the live load being absent, the probable upper value of the dead load could be a minimum of 30% greater than calculated. For a simple structure this percentage may be as follows:

10%	due to excess weight.
5%	due to misplaced reinforcement.
5%	approximation in behavior of structure.
10%	increase in stress, actual compared with calculated.
30%	Total variation assumed to occur concurrently at the section most heavily stressed.

* "Notes on Load Factor Design for Reinforced Concrete Bridge Structures with Design Applications" by Portland Cement Association, Page AB-9.

1.16.3 β (Beta) Factor

The second factor, β (beta), is a measure of the accuracy with which we can predict various kinds of loads. It also reflects the probability of one load's simultaneous application with others in a combination. It applies separately, with different magnitudes, to different loads in a combination. For example, it is usually 1.0 for dead load. It varies from 1.0 to 1.67 for live loads and impact.

Due regard has been given to sign in assigning values to beta factors, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with $\beta_D = 0.75$ are specifically included for the case where a higher dead load reduces the effects of other loads.*

The beta factors for prestressing force effects are set so that when multiplied by the respective gamma factor, the product is unity. Beta of 1.67 for live load plus impact from H loads reflects AASHTO's way of handling permit loads; the 1.00 and 1.15 for P loads on widely spaced girders accounts for bonuses sometimes granted in Caltrans' permits.

1.16.4 ϕ (Phi) Factor

ϕ (phi), the third factor, relates to materials and is called either a capacity reduction factor or a strength reduction factor. Its purpose is to account for small adverse variations in material strength, workmanship, and dimensions. It applies separately to different magnitudes for various load effects in reinforced concrete, and various manufacturing processes in prestressed concrete. Since ϕ relates to materials rather than loads, its values are given in the various material specifications. For structural steel it is almost always 1.0. For concrete it varies from 0.7 to 1.0.

* Commentary on Building Code for Reinforced Concrete (ACI 318-77), Page 33.

1.17 Load Combinations

The various load combinations to which a bridge may be subjected as well as the appropriate load factors are given in BDS 3.22, Table 3.22,1A and Table 3.22.1B.Different groups control the design of different parts of the structure, and it is often necessary to tabulate loads and effects to determine the controlling loads on members such as abutment or bent columns. It is , of course, not necessary to investigate all the loadings for a given bridge. It is often evident by inspection that only a few loadings are likely to control the design of any single type of structure.

Group I_H contains no P loads and applies to superstructures as well as substructures. Group I_{PC} is used only for P load application to superstructures with closely spaced girders where the "S-over" formulae apply.

Group I_{PW} is used for P load application to substructures and superstructures with widely spaced girders. Only one P load or one P load with one H load may be applied to the structure at a time and placed for maximum effect.

Loads as combined and factored for Service Load Design in Table 3.22.1B are for use in the service level considerations of LFD, and the rare occasions when SLD is appropriate.Stresses for the various groups are limited to the specified allowable stress for a material, adjusted by the percent overstress factors in the table.

1.18 References

Bridge Computer Manual loose leaf binder issued by Office of Structure Design, Caltrans.

Bridge Design Aids and *Bridge Design Details*, loose leaf binder issued by Office of Structure Design, Caltrans.

Bridge Design Specifications, loose leaf binder issued by Office of Structure Design, Caltrans, containing *Standard Specifications for Highway Bridges*, 13th Edition, 1983, with *Interim Specifications, Bridges, thru 1984,* published by American Association of State Highway and Transportation Officials with California modifications.

Building Code Requirements for Reinforced Concrete (ACI 318-77) with Commentary, American Concrete Institute, Detroit, December 1977.

Memos to Designers, loose leaf binder issued by the Office of Structure Design, Caltrans.

Notes on Load Factor Design for Reinforced Concrete Bridge Structures with Design Applications, Portland Cement Association, 1974.

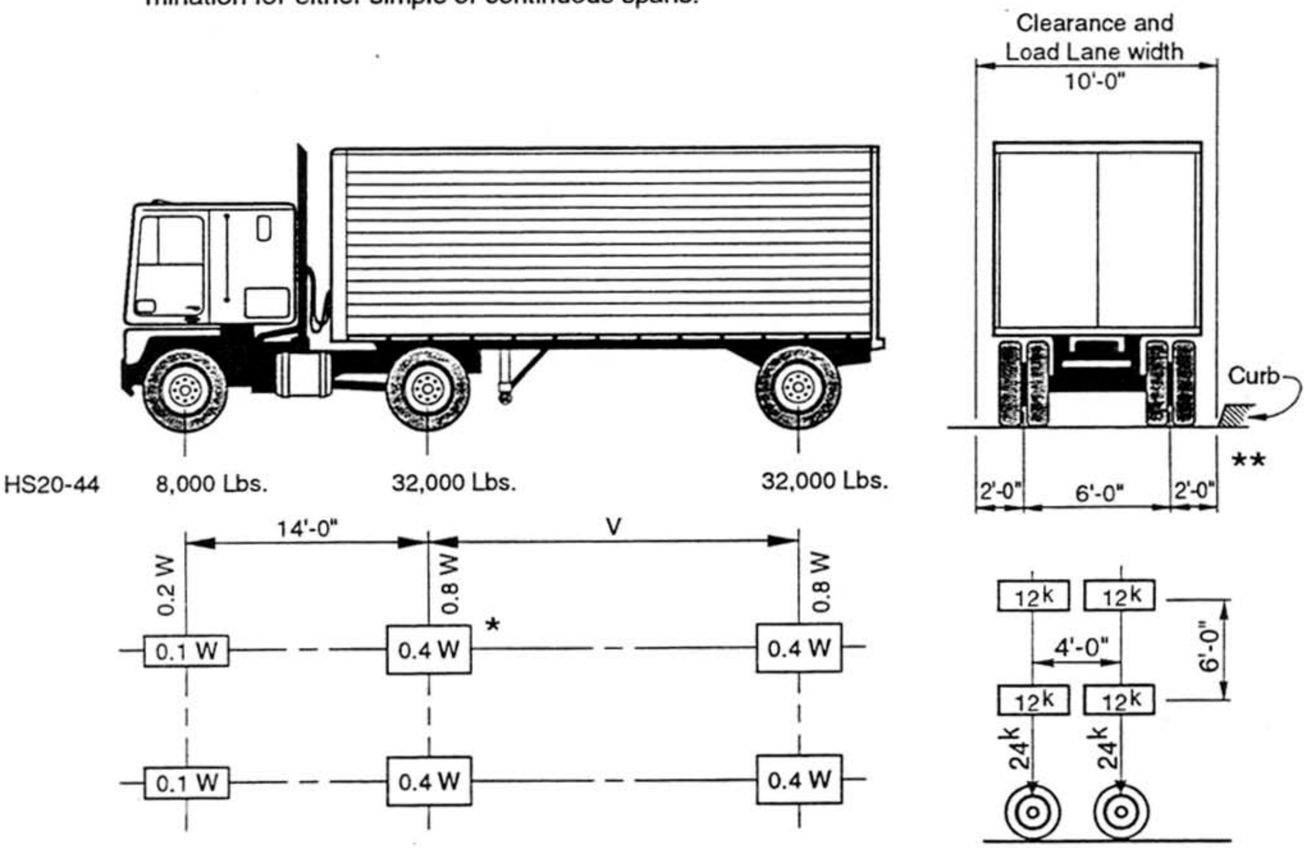
Steel Sheet Piling Design Manual, U.S. Steel

Taylor, Donald W., *Fundamentals of Soil Mechanics*, John Wiley and Sons, Inc., New York, 1948

Terzaghi and Peck, *Soil Mechanics in Engineering Practice,* John Wiley and Sons, Inc., New York, 1967.

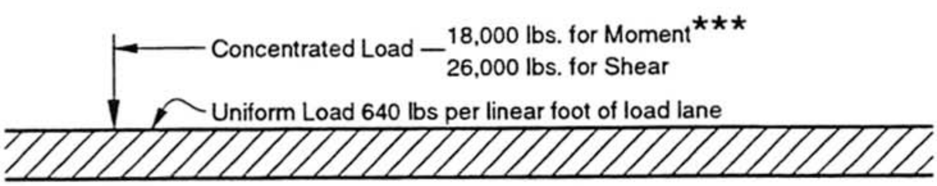
Appendix A-1

Note: Only one truck per lane is to be used for a maximum moment or shear determination for either simple or continuous spans.



ALTERNATIVE LOADING

W = Combined weight on the first two axles which is the same as for the corresponding H truck.
V = Variable spacing—14 feet to 30 feet inclusive. Spacing to be used is that which produces maximum stresses.



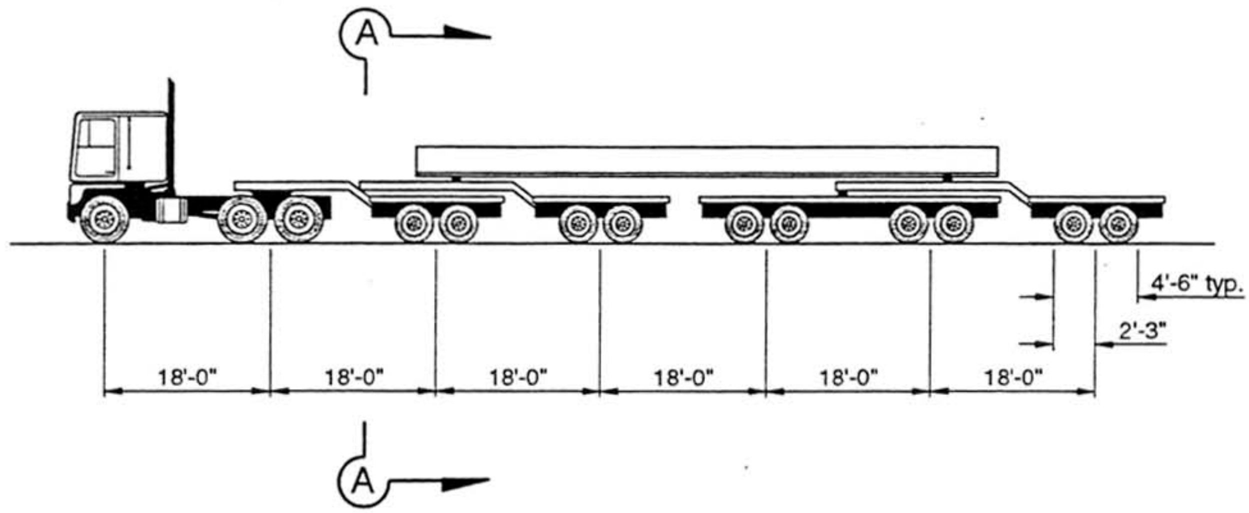
* Width of tires shall be the same as the Standard H Truck

** For slab design the centerline of wheel shall be assumed to be one foot from face of curb.

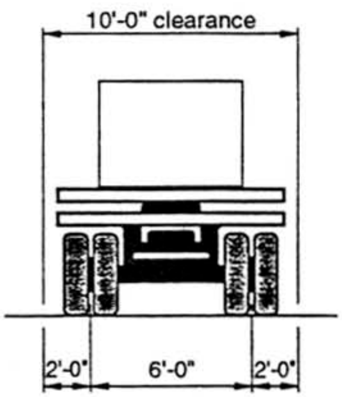
*** For continuous spans another concentrated load of equal weight shall be placed in one other span in the series, in such position as to produce maximum negative moment.

FIGURE 3

Appendix A-2



P5	26^k	48^k	48^k	–	–	–	–	Minimum Vechicle
P7	26^k	48^k	48^k	48^k	–	–	–	
P9	26^k	48^k	48^k	48^k	48^k	–	–	
P11	26^k	48^k	48^k	48^k	48^k	48^k	–	
P13	26^k	48^k	48^k	48^k	48^k	48^k	48^k–	Maximum Vechicle



Section A-A

P Loads
Permit Design Vechicles

Figure 4

Appendix A-3

Appendix A-3

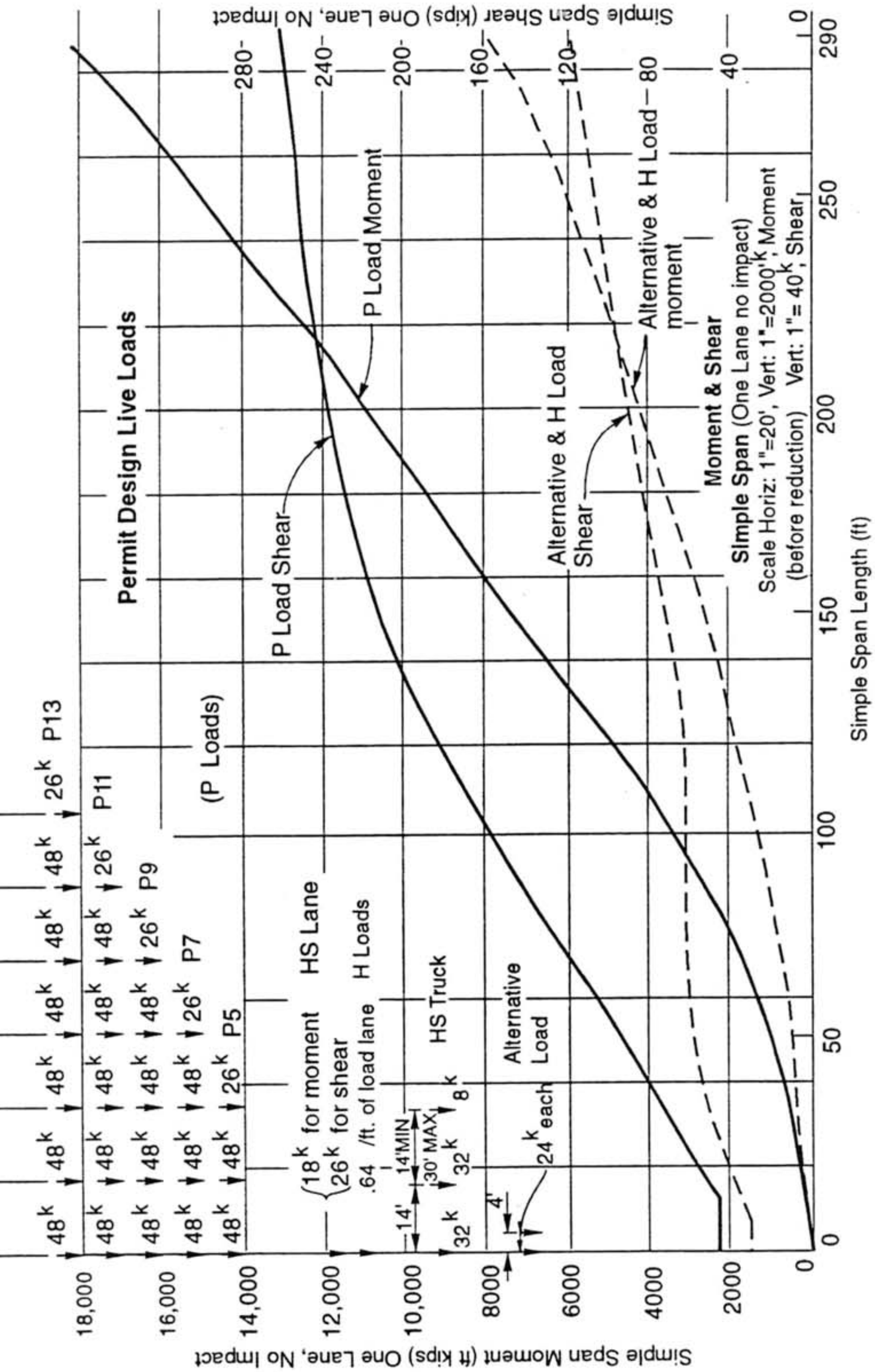


Figure 5

Appendix A-4 Example of Moment Envelope Calculations

A-4.1 Example of Moment Envelope Calculations For Continuous Structure

On continuous structures it is not always obvious by inspection how the loads should be placed to produce maximum conditions. A great deal of guesswork can be eliminated in the placing of live loads for maximum moment, shear, or reactions by the use of influence lines. The Bridge Design System will automatically generate influence lines and determine girder moments and shears along with top of column moments and reactions. It is difficult to visualize the way truck loadings actually generate a moment or shear envelope, so we will go through a step-by-step development of these envelopes.

This example treats only application of H loads. The vehicle is moved across the structure from left to right and from right to left. The moments and shears at the various tenth points of the spans are noted as this movement occurs. The envelope is determined when the moment or shear at each tenth point is maximum. Table A-4 shows a summary of this work for maximum moment. Table A-5 shows maximum conditions for shear. The maximum moments or shears are shown at the tenth points along with the position of the vehicle or lane load that produces the maximum condition.

Bridge Design System automatically moves the applicable live loads across the structure in both directions, notes the values of moment and shear and reports the maximums in tabular or graphical form at each tenth point.

We will use a simplified three span continuous structure with 50-foot spans simply supported (Figure 6).

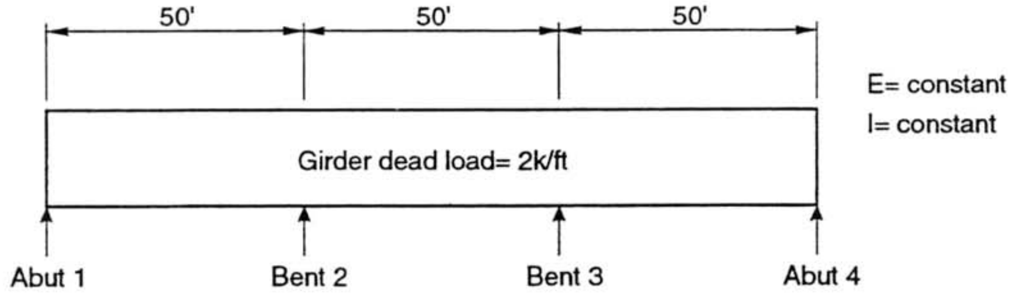


Figure 6

For purposes of this example, assume that HS 20 live loading consists of 1.5 wheel line or 0.75 lane per girder. Impact is 28.6%

The (LL+I)H values given in Tables A-4 and A-5 result from applying HS 20 loads converted as follows:

Truck Load

$$\frac{8\text{ kip}}{\text{lane}}\times\overset{\text{Impact}}{\frac{1{,}286}{1}}\times\frac{1.5\text{ wheel line}}{\text{girder}}\times\frac{1\text{ lane}}{2\text{ wheel line}}=7.7\,\text{K/girder}$$

$$\frac{32\text{ kip}}{\text{lane}}\times\frac{1.286}{1}\times\frac{1.5\text{ wheel line}}{\text{girder}}\times\frac{1\text{ lane}}{2\text{ wheel line}}=30.9\,\text{K/girder}$$

Lane Load

$$\frac{0.640\text{ Kip}}{\text{ft. lane}}\times\frac{1.286}{1}\times\frac{1.5\text{ wheel line}}{\text{girder}}\times\frac{1\text{ lane}}{2\text{ wheel line}}=0.617\,\text{K/girder/ft.}$$

$$\frac{18\text{ Kip}}{\text{lane}}\times\frac{1.286}{1}\times\frac{1.5\text{ wheel line}}{\text{girder}}\times\frac{1\text{ lane}}{2\text{ wheel line}}=17.4\,\text{K/girder}$$

Maximum moment and shear envelopes are determined similarly when P loads are applied.

The values of (LL+I)P moments for 0.75 lane of P loads are given for comparison purposes on Table A-4, line 21. Similar values for shear are given in Table A-5, line 13.

It should be noted that even though these raw values for P loads are higher than the H load results, the application of different load factors for each brings the results closer together during design.

A-4.2 Dead Load Moments (Table A-4, Lines 2 and 3)

Dead load moments are given by Bridge Design System using the uniform load of 2 kips per foot. A plot of these values closely follows parabolic curves.

A-4.3. Envelope Curve For Positive Girder Moment in Span 1 (Table A-4, Lines 4-11)

For the 50-foot span, the HS truck produces maximum positive moment in the span. Each horizontal line of values indicates the moments at the tenth points of the first two spans of the three span structure. These moments are caused by the 0.75 lane plus impact for the HS truck located as shown by the circles indicating axles. The smaller circle represents the front axle of the truck.

While not always true, the maximum moment due to two or more moving concentrated loads generally occurs when the heaviest load is at the section.

From the tabulation it can be seen that the maximum positive moment is at the 0.4 point of the span (Table A-4, line 7). For practical purposes the truck could be placed anywhere from the 0.35 to the 0.45 point to obtain the maximum positive moment, without appreciable error.

For the condition of dead load and live load indicated, the tabulation of underlined (LL+I)H moments when added to the dead load moments gives the maximum positive moment that can be obtained at each of the points shown (line 19). A curve passed through these points constitutes an envelope curve pf positive moment (see Figure 7).

A-4.4 Envelope Curve For Negative Girder Moment (Table A-4, Lines 12-15)

Maximum negative (LL+I)H moment over the support (Bent 2) is produced by the lane load in Spans 1 and 2 with concentrated loads for moment (line 12). Moments produced by the lane loading are slightly greater than those produced by the HS 20 truck.

Except near Bent 2, maximum live load negative moment in Span 1 is produced by loading Span 2. It is noted that a plot of the Span 1 moments will produce a straight line. The load position is shown on line 13. The negative moment in unloaded spans is frequently overlooked by beginners.

The envelope curve for negative live load moment in Span 2 is obtained by loading the adjacent span as shown on line 14.

An additional refinement to the negative moment envelope is obtained by loading Spans 1 and 3 with the lane loading shown on line 15. This loading slightly modifies the envelope curve between the 0.4 and 0.5 points on Span 2. However, the modification is of little practical value in determining cutoff of negative bars in a concrete span, because the value falls below the resisting moment of longitudinal bars which are normally carried through.

The envelope curve for negative moment is obtained by combing maximum values of dead load and (LL+I)H. The values are tabulated on line 20 and the envelope is plotted in Figure A-4. It is seen that the positive and negative moment envelopes overlap along the base line. This is characteristic of continuous structures.

A-4.5 Envelope Curve For Positive Girder Moment In The Interior Span Of Three Continuous Spans (Table A-4, Lines 16-17)

The truck load again produces maximum positive center span live load moments. The moments due to truck loadings at the 0.2 and 0.5 points in the span are shown. These points are control points which determine the envelope cure which approximates a 2nd degree parabola. The combined values of dead and live load moments are shown on line 19.

Appendix A-5 Example of Shear Envelope Calculations

A-5.1 Envelope Curve For Girder Shear In End Span Of Three Continuous Spans (Table A-5, Lines 4-7)

The same three-span continuous structure is utilized to illustrate the placing of load in continuous spans for determination of live load shear. Dead load shears are tabulated on Table A-5 line 3 and plotted on Figure A-5.

It can be found that the maximum live load shear at Abutment 1 is governed by the truck loading.

For practical utilization of the curve of maximum shear it is necessary to determine the LL+I shear at only three points; at Abutment 1, at the 0.4 point of Span 1, and left of Bent 2. These points are connected with straight lines as indicated in the diagram on Table A-5.

The variation between the actual shear envelope which has a slight curve and the straight line method described above is of no practical concern, as other empirical assumptions may introduce a greater difference.

The shear diagram for use in determining stirrup spacing in concrete beams, or stiffener spacing in plate girders is constructed by combining the dead load shears with the maximum (LL+I)H shears as shown on Figure A-5.

A-5.2 Envelope Curve For Girder Shear In Center Span Of Three Continuous Spans (Table A-5, Lines 8-9)

In this structure the DL+(LL+I)H shear curve for Span 2 is symmetrical about the centerline of span; therefore it is necessary to compute shear at right of Bent 2 and at the centerline only. The load positions and values of live load shear for Span 2 are given on lines 8 and 9. Negative shear in Span 2 has the same value as postive shear and is determined by placing the truck on the other end of the span.

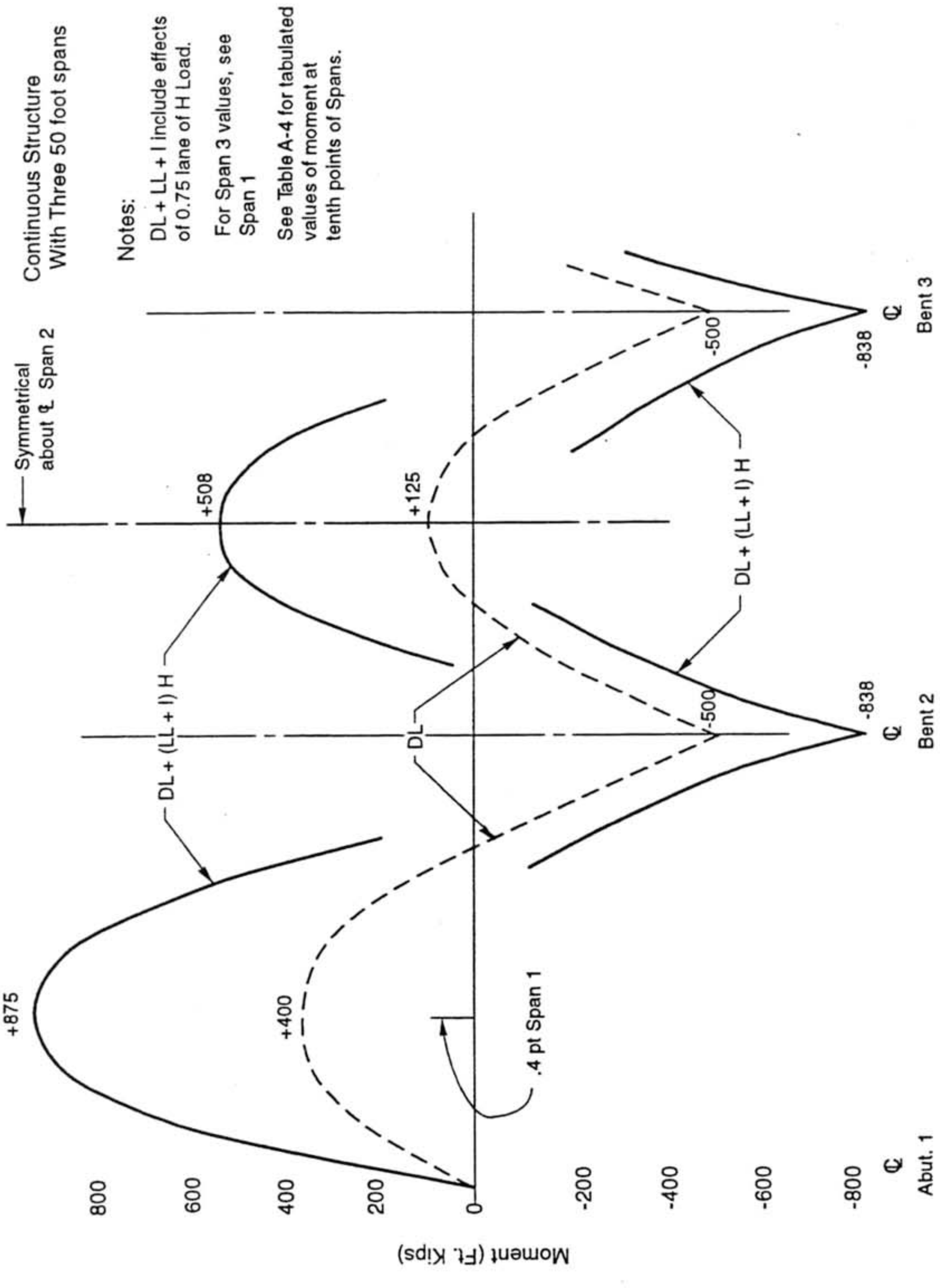
The (LL+I)H values are added to the dead load shears to give the DL+(LL+I)H curve shown in Figure A-5.

Subsequent sections of the course will cover the utilization of shear and moment envelopes for the design of various types of structures.

Moment, Table A-4

8^K/lane $(1.268)\left(1.5\frac{\text{wheel line}}{\text{girder}}\right)\left(\frac{\text{lane}}{\text{2 wheel line}}\right) = 7.7^K$/girder ○ $32^K(1.286)\left(\frac{1.5}{2}\right) = 30.9^K$/girder ○

	10		20		30		40		50		10		20		30		40		50	Distance Along Span (ft.)	1
girder dead load= 2 K/1																				Dead Load (Uniform l)	2
175	300	375	400	375	300	175	0	-225	-500	-275	-100	25	100	125	100	25	-100	-275	-500	DL Moments (Ft. Kips)	3
(LL+I)H Loads with Associated Moments (Ft. Kips)																					
226	298	370	412	329	247	150	29	-92	-213	-186	-160	-133	-106	-80	-53	-27	0	27	53		4
187	373	405	437	439	317	195	57	-103	-264	-231	-198	-165	-132	-99	-66	-33	0	33	66		5
149	298	446	441	436	399	239	80	-95	-294	-257	-220	-184	-147	-110	-73	-37	0	37	73	Envelope Curve of Positive Moment in Span 1	6
146	261	368	475	428	381	303	102	-100	-301	-263	-226	-188	-150	-113	-75	-38	0	38	75		7
179	358	413	437	462	332	202	65	-104	-272	-238	-204	-170	-136	-102	-68	-34	0	34	68		8
142	283	425	443	430	418	251	84	-91	-297	-260	-222	-185	-148	-111	-74	-37	0	37	74		9
139	254	354	454	431	377	322	114	-95	-303	-265	-228	-190	-152	-114	-76	-38	0	38	76		10
104	209	290	356	422	364	276	188	-55	-298	-261	-223	-186	-149	-112	-74	-37	0	37	74		11
.617 K/1				17.4 K			.617 K/1					17.4 K		.617 K/1							
72	129	171	197	208	182	76	-47	-185	-338	-184	-46	76	126	131	120	94	53	-4	-76	Envelope of Negative Mom. @ Bent 2	12
-23	-47	-70	-93	-117	-140	-164	-187	-210	-234	-24	186	272	327	383	284	185	78	-59	-196	Envelope of Negative Mom. @ Span 1	13
127	243	331	419	437	370	304	136	-84	-305	-267	-229	-191	-152	-114	-76	-38	0	38	76	Envelope of Negative Mom. @ Span2	14
.617 K/1				17.4 K																.617 K/1	
92	168	229	275	305	299	211	108	-10	-144	-144	-144	-144	-144	-144	-144	-144	-144	-144	-144		15
-23	-46	-70	-93	-116	-139	-162	-186	-209	-232	-15	202	265	328	360	269	177	70	-60	-190	Control Points for Positive Moment in Span 2	16
-23	-47	-70	-93	-117	-140	-164	-187	-210	-234	-24	186	272	327	383	284	185	78	-59	-196		17
226 -23	373 -47	446 -70	475 -93	462 -117	418 -140	322 -164	188 -187	74 -210	76 -338	70 -267	202 -229	308 -191	372 -152	383 -144						(LL+I)H Envelope of Max. Mom. for .75 Lane of H Loads	18
401	673	821	875	837	718	497	188				102	333	472	508						Positive Mom. DL+(LL+I)H	19
							-187	-435	-838	-542	-329	-166	-52	-19						Negative Mom. DL+(LL+I)H	20
330 -32	522 -65	603 -97	661 -129	675 -161	579 -194	438 -226	255 -258	102 -417	114 -789	95 -445	277 -341	417 -284	502 -227	526 -170						(LL+I)P Envelope of Max. Mom. for .75 Lane of P Loads	21



Moment Envelope
Figure 7

Shear, Table A-5

		0		10		20		30		40		50		10		20		30		40		50
Distance Along Span (ft.)	1	0		10		20		30		40		50		10		20		30		40		50
Dead Load (Uniform l)	2	girder dead load= 2 K/1																				
DL Shears (Kips)	3	40	30	20	10	0	-10	-20	-30	-40	-50	-60	50	40	30	20	10	0	-10	-20	-30	-40	-50
(LL+I)H Loads with Associated Moments (Ft. Kips)																							
Shear at Abutment 1	4	54	23	23	-8	-8	-8	-16	-16	-16	-16	-16	4	4	4	4	4	4	4	4	4	4	4
Positive Shear at .4 pt. Span 1	5	23	28	23	23	23	-8	-8	-39	-39	-39	-47	8	8	8	8	8	8	8	8	8	8	8
Negative Shear at .4 pt. Span 1	6	42	42	11	11	11	-20	-20	-20	-20	-20	-20	5	5	5	5	5	5	5	5	5	5	5
Shear at Left of Bent 2	7	9	9	9	9	9	2	2	2	-29	-29	-60	5	5	5	5	5	5	5	5	5	5	5
Shear at Right of Bent 2	8	-3	-3	-3	-3	-3	-3	-3	-3	-3	-3	-3	57	26	26	-4	-4	-4	-12	-12	-12	-12	-12
Positive Shear at .5 pt. Span2	9	-3	-3	-3	-3	-3	-3	-3	-3	-3	-3	-3	21	21	21	21	21	21	-10	-10	-41	-41	-41
(LL+I)H Envelope of max. Shear for .75 Lane of H loads	10	54 -5	45 -5	37 -9	30 -13	23 -20	16 -27	11 -34	7 -42	4 -49	2 -55	2 -60	57 -8	51 -8	43 -8	36 -11	28 -14	21 -21	14 -28	11 -36	8 -43	8 -51	8 -57
Positive Shear DL+(LL+I)H	11	94	75	57	40	23	6						107	91	73	56	38	21	4				
Negative Shear DL+(LL+I)H	12				-3	-20	-37	-54	-72	-89	-105	-120					-4	-21	-38	-56	-73	-91	-107
(LL+I)P Envelope of max. Shear for .75 Lane of P loads	13	81 -7	66 -7	52 -12	40 -17	30 -25	21 -39	13 -52	6 -63	2 -77	2 -90	2 -103	98 -11	85 -11	73 -11	59 -11	47 -21	34 -34	21 -47	11 -59	11 -73	11 -85	11 -98

℄ abut. 1 ℄ bent 2 ℄ bent 3

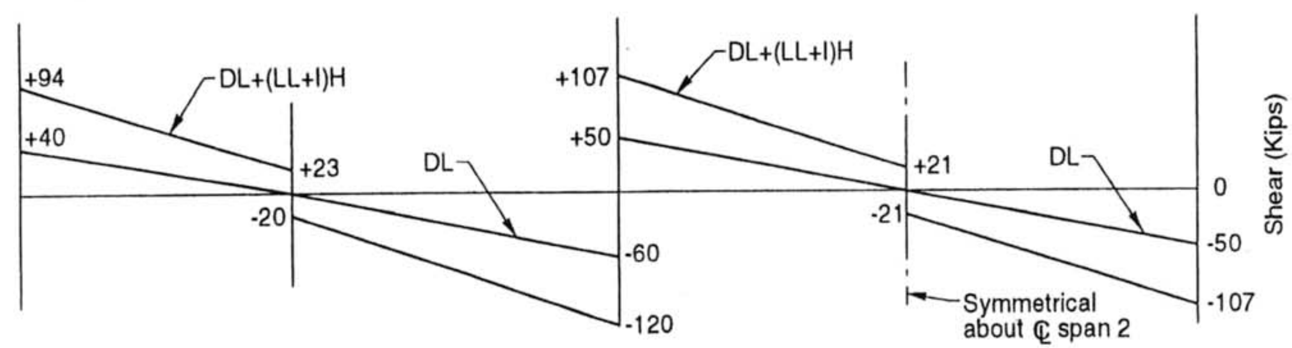


Figure A-5
Shear Envelope

DL+(LL+I)H
3-50 foot span
continuous structure
Note: For span 3 values see span 1.